Technical Appendices

Appendix F – Geology and Soils

Twining Laboratories, Inc. Preliminary Geotechnical Investigation, Proposed Retail Shopping Center, Northeast Corner of State Highway 101 and Cochrane Road, Morgan Hill, California. November 1, 2004.

THE TWINING LABORATORIES, INC.
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ANALYTICAL CHEMISTRY • ENVIRONMENTAL SERVICES
GEOTECHNICAL ENGINEERING • SAMPLING SERVICES
CONSTRUCTION INSPECTION & MATERIALS TESTING

DRAFT

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED RETAIL SHOPPING CENTER

NORTHEAST CORNER OF STATE HIGHWAY 101 AND COCHRANE ROAD

MORGAN HILL, CALIFORNIA

Project Number: A07261.03-01

For:

Browman Development Company, Inc.
100 Swan Way, Suite 206
Oakland, CA 94621-1459

November 1, 2004

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November 1, 2004 A07261.03-01

Browman Development Company, Inc.
100 Swan Way, Suite 206
Oakland, CA 94621-1459

Attention: Mr. Jerry Neighbors

Subject: *DRAFT*; Preliminary Geotechnical Investigation
Proposed Retail Shopping Center
Northeast Corner of State Highway 101 and Cochrane Road
Morgan Hill, California

Dear Mr. Neighbors:

We are pleased to submit this preliminary geotechnical engineering investigation report prepared for the general project planning of the proposed retail shopping center development to be located at the northeast corner of the intersection of State Highway 101 and Cochrane Road in Morgan Hill, California. The contents of this report include the purpose of the study, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations.

Grading, architectural, structural plans, etc. for the proposed development were not available at the time of this study. It is recommended that those portions of the plans and specifications that pertain to earthwork, pavements, foundations and other related drawings be reviewed by The Twining Laboratories, Inc. (Twining), when they become available, to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement, however, it is recommended that the client provide these documents for our review prior to their issuance for construction bidding purposes. This report presents preliminary information relative to the proposed development. As such, additional geotechnical engineering investigations will be required subsequent to preparation of final project plans.

In addition, it is recommended that Twining be retained to provide inspection and testing services for the excavation, earthwork, pavement, and foundation phases of construction. These services are necessary to determine if the subsurface conditions are consistent with those used in the analyses and formulation of recommendations for this investigation, and if the pertinent aspects of construction comply with our recommendations. These services are not, however, part of this current contractual agreement. We would appreciate the opportunity to provide a proposal for these additional services after construction documents are completed. A representative of our firm (800-268-7021) will contact you in the near future regarding these services.

CORPORATE
2527 Fresno Street
Fresno, CA 93721-1804
(559) 268-7021
Fax 268-7126

MODESTO
5253 Jerusalem Court, Suite E
Modesto, CA 95356-9322
(209) 342-2061
Fax 579-1480

VISALIA
130 North Kelsey St., #H6
Visalia, CA 93291-9000
(559) 651-8280
Fax 651-8288

BAKERSFIELD
3651 Pegasus Drive, #117
Bakersfield, CA 93308-6843
(661) 393-5088
Fax 393-4643

MONTEREY
501 Ortiz Avenue
Sand City, CA 93955
(831) 392-1056
Fax 392-1059

SACRAMENTO
5675 Power Inn Road, Suite C
Sacramento, CA 95824
(916) 381-9477
Fax 381-9478

We appreciate the opportunity to be of service to Browman Development Company, Inc. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
THE TWINING LABORATORIES, INC.

DRAFT

Harry C. Wise
Monterey Area Engineering Manager
Geotechnical Engineering Division

HCW

EXECUTIVE SUMMARY

The Twining Laboratories, Inc. (Twining) was authorized by Mr. Jerry Neighbors with Browman Development Company, Inc. to conduct a preliminary geotechnical engineering investigation for the proposed retail shopping center development, as outlined in our proposal dated August 24, 2004. The subject property is located at the northeast corner of the intersection of State Highway 101 and Cochrane Road in Morgan Hill, California and comprises approximately 56 acres. The purpose of our study was to provide preliminary geotechnical engineering parameters for use in design of foundations, pavement sections, slabs-on-grade, and preparation of related construction documents.

Based on a preliminary site plan provided by the Browman Development Company, Inc., the proposed construction will include the following structures:

- Nine (9) Retail Store Buildings (8 Major Tenants and 1 Marketplace) will comprise approximately 492,943 square feet in plan dimensions;
- One (1) Store Area will comprise approximately 14,000 square feet in plan dimensions; and
- Eight (8) Pads will comprise approximately 44,500 square feet in plan dimensions.

It is anticipated that the proposed larger Major Stores and other smaller buildings (pads) will consist of single story structures with concrete masonry unit, steel, wood, or concrete tilt-up walls, steel and/or wood frame roofs, and concrete slab-on-grade floors. Basements are not anticipated, however, depressed truck ramps and loading docks typically accompany the major stores. The project will also include asphaltic concrete and Portland Cement Concrete (PCC) parking and drive areas.

The scope of work provided in this report does not meet the geotechnical report requirements for major retail store tenants, or include any recommendations for off-site street improvements. It is anticipated that the building column and line loads will be moderately variable for the various structures constructed within the proposed shopping center. In any case, for the purposes of this geotechnical engineering investigation, maximum column and line loads of 150 kips and 5 kips per lineal foot, respectively, have been assumed for the proposed development.

It should be noted that once a definitive site plan with building types is developed, design level geotechnical engineering investigation reports will be required to address major store requirements, specific design building loads of the various structures, and building pad earthwork requirements for the different loading and subsurface site conditions.

At the time of the field exploration, the project site was predominantly used for agricultural purposes. In addition, two (2) residential dwellings and seven (7) additional buildings were present at the time of the field exploration. The seven additional buildings included barns, a garage, tack rooms, pump houses, and restrooms. On-site septic systems were reported by the landowners in association with the residential dwellings. Also, domestic wells and irrigation wells were noted on the site.

EXECUTIVE SUMMARY (con't)

During the geotechnical field exploration conducted on September 1 and 2, 2004, a total of twelve (12) test borings were drilled within the proposed site development. Both disturbed and undisturbed soil samples were collected and returned to our laboratory for classification and testing.

In two (2) borings (B-3 and B-11) of the twelve (12) borings drilled, near surface, very stiff, sandy lean clays were encountered from the ground surface to depths of about 3 feet below site grade (BSG). However, in general, the near surface soils encountered at the boring locations consisted of very stiff to hard sandy silts interbedded with medium dense to dense silty sands extending from the ground surface to depths of about 3 feet BSG. From about 3 feet to about 20 feet BSG, the very stiff to hard silts and medium dense to dense silty sands were interbedded with medium dense to very dense silty to clayey gravels. Below 20 feet BSG, the interbedded hard silts and medium dense to very dense sands and gravels extended to the maximum depth explored of 51½ feet BSG, with the exception of one (1) boring (B-8) where very stiff to hard sandy clay was encountered between depths of 20 and 25½ feet BSG.

The near surface soils exhibited high compressibility and high collapse potential, high shear strength, and poor to fair pavement support characteristics. Near surface sandy lean clay soils were encountered in two (2) of the twelve (12) borings. The results of two (2) Atterberg Limits tests indicated that the near surface sandy lean clays had liquid limits of 36 and 21, and plasticity indices of 21 and 5, respectively. Also, the near surface sandy clay to sandy silt soils exhibited a low potential for expansion (EI=11 and 23).

The chemical analyses results indicate a resistivity value of 16,000 ohms/centimeter and a pH value of 5.8. Based on the resistivity value, the soils exhibit "mildly corrosive" corrosion potential. In addition, the results of soil sample analyses indicate sulfate concentrations of 0.00068 by percent weight. Therefore, a low potential for sulfate attack on concrete placed in contact with the soils is anticipated.

From a geotechnical standpoint, the site is suitable for the proposed construction with regard to support of shallow spread foundations and concrete slabs-on-grade, provided the recommendations contained in this preliminary report are followed. The primary geotechnical concerns are: 1) the presence of undocumented fill soils; 2) the presence of disturbed, highly compressible and collapsible, native, near surface soils encountered from the ground surface extending to about 3 feet BSG; 3) the potential to incorporate over-sized material (gravel and cobbles) within the footing zone; and 4) the potential to encounter over-sized materials (gravel and cobbles) during grading operations.

Groundwater was not encountered in the test borings drilled at the time of the field exploration (September 1 and 2, 2004). Review of maps prepared by the Department of Water Resources (DWR) indicates that there are no wells in the nearby Morgan Hill area that provides groundwater elevations. Groundwater was not encountered in any of our borings, with the deepest boring being drilled to 51½ feet.

EXECUTIVE SUMMARY (con't)

The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest known active or potentially active fault is the Calaveras Fault (South of the Calavaras Reservoir segment), located about 3 miles (4¾ km) east of the site. Therefore, the potential for fault rupture at the site is considered low.

Based on the site topography, depth to groundwater below the site, and the high resistance to standard penetration testing (N values), the potential for liquefaction to impact the site due to loss of foundation bearing capacity, surface manifestations, and lateral spreading is considered low. Considering the shallow depth to dense to very dense gravel soils, total seismic settlement of not more than 1/4-inch would be expected to occur under shaking from the design-basis earthquake (0.89g and a magnitude of 7.9). Considering the size of the proposed structures, the recommended site preparation, and the soil conditions encountered, a differential seismic settlement of ¼-inch in 40 feet should be anticipated for design.

However, it should be noted that the northern portion of the site (approximately ¼ of the site) is located within an area of potential liquefaction as identified by the CGS on the State of California Seismic Hazard Zones, Morgan Hill Quadrangle, dated April 17, 2004, available on their website at (http://gmw.consrv.ca.gov/shmp/download/pdf/pzn_morgh.pdf). The map shows areas where historical occurrence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693©) would be required. However, based on the results of the preliminary evaluation included herein, the risk of liquefaction at this site is considered low. Due to the potential variability of the subsurface soils and depth to groundwater across the site, it is recommended that the proposed structures be evaluated on a case by case basis as part of future design level geotechnical engineering investigations.

Over-excavation and placement of engineered fill below foundations are recommended to limit static settlements to 1 inch total and ½ inch differential over a horizontal distance of 40 feet. Site preparation should include stripping and removal of the existing structures, trees, grasses, organic debris, as well as over-excavation and placement of engineered fill below foundations, slabs and flatwork areas. Stripping should be conducted prior to over-excavation. Stripping depths should be sufficient to remove trees, vegetation, organic matter, etc. Limbs, tree branches, roots, etc. should not be disced into the near-surface soils. These materials should be raked and hand-picked, as necessary, to ensure proper removal. For estimating purposes, a minimum stripping depth of 6 inches should be used for the site in general. However, it should be anticipated that tree roots exceeding ¼ inch in diameter will extend below the minimum stripping depth.

The proper removal of trees and their associated root structures is an import aspect of this project and should be properly planned and monitored. A demolition plan should be developed by the contractor and should include a survey of the site. The plan should specify how the contractor

EXECUTIVE SUMMARY (con't)

proposes to remove the building structures, septic tanks and leach lines,. irrigation lines and other underground utilities, trees, roots, and organic matter generated during the removal process and how the excavations and loose soils generated during this process will be addressed.

ant

The depth of excavations should be sufficient to remove roots larger than ¼ inch in diameter. It is anticipated that roots, root balls, and loose soils and voids resulting from tree removal operations will extend to depths on the order of 3 to 4 feet BSG. Therefore, a minimum excavation depth of 4 feet is estimated to be required to remove roots larger than 1/4 inch in diameter for the trees. The actual depth of stripping should be reviewed by Twining at the time of construction based on loss-on-ignition tests conducted to determine organic content in the soil (note: maximum limit of 3 percent organics by weight). Deeper excavations may be required in localized areas to remove tree roots.

It is assumed that the proposed construction can tolerate total and differential static settlements of approximately 1-inch and ½-inch, respectively. There are several options available to reduce the estimated total and differential static settlements to 1-inch and ½-inch in 40 feet, respectively. Over-excavation of the near surface soils and replacement of these soils as engineered fill is one of the more common methods of dealing with compressible soils. To reduce the differential static settlements from the static loads to within ½-inch in 40 feet, the on-site soils beneath the proposed structures and for a distance of 10 feet beyond the perimeter of the structures should be over-excavated to provide a minimum of 24 inches of engineered fill beneath the proposed foundations and slab-on-grade floor areas, at least 36 inches of engineered fill below preconstruction site grades, or at least 12 inches of engineered fill below improvements to be removed, whichever is greater. In addition, the building pad area, including the over-build zone 10 feet beyond the building perimeter, should be over-excavated to the same depth that is required for over-excavation underneath the footings. Exterior slab-on-grade floor areas and pavement areas should be underlain by engineered fill extending to at least 12 inches below the slab or pavement, at least 18 inches below preconstruction site grades, or at least 12 inches below improvements to be removed, whichever is greater. Improvements to be removed for building pads, exterior slabs-on-grade, and pavement areas include the two residential structures and their associated barns, sheds, garages, septic tanks and leach lines, as well as any irrigation lines or other underground utilities that may exist on site. It may be more economical to backfill with engineered fill areas with improvements to be removed prior to conducting over-excavation for building pads, exterior slab-on-grade floor areas, or pavement areas. Exterior flatwork should be underlain by a minimum of 12 inches of granular non-expansive soils which include a minimum of four (4) inches of aggregate base directly below the slab-on-grade.

One of the primary geotechnical concerns is the potential to encounter oversized, greater than 3 inches in overall diameter, gravel and cobble material during the grading operations (from the surface to depths of about 7 feet BSG). If these larger materials are encountered during the over-excavation for the building pads, they should be broken up/crushed into fragments that are three (3) inches or less in size along the largest dimension. If the cobbles cannot be broken up into fragments

EXECUTIVE SUMMARY (con't)

that are three (3) inches or less in size along the largest dimension, the contractor should remove the over sized fragments from the fill material by screening or other techniques approved by Twining. The screening activity would be anticipated to increase the costs of the earthwork Neatly cut footings may be difficult to obtain due to the presence of oversized materials.

In order to better determine the requirements for screening of the on-site materials, it is recommended test pits be excavated at this site under the direction of Twining. In addition, potential grading, excavating and underground contractors should be present during these explorations to make their own observations of the gravels and cobbles in the soils exposed in the test pits. This activity will assist the contractors in preparation of bids. It has been reported by one of the property owners that fill soils were hauled into the site as a part of an adjacent pipeline project. However, the extent of fill soils was not discernable upon visual observation of the surface features of the site nor review of a low elevation aerial photograph. Therefore, as part of the backhoe exploration, the potential presence and limits of onsite fill soils (if any) can be further evaluated.

Footings should have a minimum depth of 18 inches (note: 24 inches for major stores and two-story structures) and a minimum width of 12 inches (note: 15 inches for major stores and two-story structures), regardless of load. The foundations may be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot for dead plus live loads assuming compliance with the site preparation earthwork recommendations presented in this report.

Preliminary asphaltic concrete pavement and Portland cement concrete sections are presented in the recommendations section of this report. The structural sections were designed using the gravel equivalent method in accordance with Chapter 600 of the California Department of Transportation Highways Design Manual (fourth edition). The structural vehicle loadings were based on typical traffic loadings for this type and size of facility. The analyses were based on a range of traffic index values from 5.0 to 8.0. Traffic indices for the project should be selected by the project Civil Engineer or as required by the applicable tenants. If the pavements are placed prior to construction, the additional construction traffic should be considered in the selection of the design traffic index. If more frequent truck traffic is anticipated than that indicated by the maximum traffic index, Twining should be contacted to re-evaluate the traffic index values.

Based on the ASTM Special Technical Publication 741 and the analytical results of one (1) soil sample analysis, the soil has a "mildly corrosive" corrosion potential to ferrous alloy pipes, as indicated by a resistivity value of 16,000 ohms/centimeter and a pH value of 5.8. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on the "mildly corrosive" corrosion potential of the soil. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated.

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED RETAIL SHOPPING CENTER

NORTHEAST CORNER OF STATE HIGHWAY 101 AND COCHRANE ROAD

MORGAN HILL, CALIFORNIA

Project Number: A07261.03-01

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical engineering investigation for the general project planning for the proposed retail shopping center development to be located at the northeast corner of the intersection of State Highway 101 and Cochrane Road in Morgan Hill, California. The Twining Laboratories, Inc. (Twining) was authorized by Mr. Jerry Neighbors with Browman Development Company, Inc., to conduct this preliminary geotechnical engineering investigation, as outlined in our proposal dated August 24, 2004.

The contents of this report include the purpose of the study and the scope of services provided. The site history, previous studies, existing site features, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The three report appendices contain the drawings (Appendix A), the logs of borings (Appendix B), and the results of laboratory tests (Appendix C).

The Geotechnical Engineering Division of Twining, headquartered in Fresno, California, performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The purpose of this study was to conduct a field exploration and laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following information for preliminary design purposes:

2.1.1 Geotechnical parameters for use in design of foundations and slabs-on-grade, and development of lateral resistance;

2.1.2 Recommendations for site preparation including over-excavation, fill placement, moisture conditioning, and compaction of engineered fill soils;

2.1.3 Recommendations for the design and construction of asphaltic concrete and Portland Cement Concrete (PCC) pavements;

2.1.4 Recommendations for temporary excavations and trench backfill; and

2.1.5 Conclusions regarding soil corrosion potential.

This report is provided specifically for the Browman Development Company, Inc. for the proposed retail development referenced in the Anticipated Construction section of this report.

This investigation did not include a flood plain investigation, compaction tests, environmental investigation, or environmental audit.

2.2 **Scope:** Our proposal, dated August 24, 2004, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 A preliminary site plan for the proposed retail development, provided by Browman Development Company, Inc., dated July 28, 2003, was reviewed.

2.2.2 A low elevation aerial photograph of the site was reviewed.

2.2.3 A preliminary draft of a report entitled "Phase I Environmental Site Assessment; Northeast Corner of State Highway 101 and Cochrane Road, Morgan Hill, California," herein referred to as the preliminary Environmental Report (our reference A07261.01), dated June 11, 2004, prepared by Twining, was reviewed.

2.2.4 The State of California Seismic Hazard Zone map of the Morgan Hill Quadrangle, dated April 17, 2004, was reviewed.

2.2.5 Site reconnaissance, drilling test borings, and soil sampling were conducted.

2.2.6 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils.

2.2.7 Mr. Jerry Neighbors (Browman Development Company, Inc.) and Mr. Rowan C. Punchihewa (JP DiNapoli Companies, Inc.) were consulted during the investigation.

2.2.8 The data obtained from the investigation were evaluated to develop an understanding of the subsurface conditions and engineering properties of the subsurface soils.

2.2.9 This preliminary geotechnical engineering investigation report was prepared to present the purpose and scope, background information, field exploration procedures, findings, evaluation, conclusions, and recommendations.

3.0 BACKGROUND INFORMATION

The site history, previous studies, existing site features, and the anticipated construction are summarized in the following subsections.

3.1 Site History: According to the preliminary Environmental Report, the project site has been used for agricultural purposes since at least the 1910's, with the exception of scattered rural residences on the site. Based on conversations with Ms. Jean Millard-Low, property owner of one of the on-site parcels (APN 728-37-001), imported fill soil, generated during construction of a Santa Clara Water District pipeline project approximately one mile north of the site, was placed throughout the site. The preliminary Environmental Report states it is believed that the fill soil was derived from rangeland and agricultural land.

3.2 Previous Studies: The preliminary Environmental Report, prepared by The Twining Laboratories, Inc., for the site, was reviewed. No other previous geotechnical engineering, geological, or environmental studies conducted for this site were provided for our review. If these documents are, or become available, they should be provided to Twining for review.

3.3 Site Description: The project site is an approximate 56-acre area located on the northeast corner of the intersection of State Highway 101 and Cochrane Road in Morgan Hill, California. A site location map is presented as Drawing No. 1 in Appendix A of this report. The site is bound to the north by agricultural land which includes orchards, greenhouses, and barns; to the east by the agricultural land including orchards; to the west by a canal and Highway 101,and commercial property beyond; and to the south by Cochrane Road with single family residences and agricultural land beyond. According to the 7½ minute series topographic map (Morgan Hill, California), produced by the United States Geological Survey (USGS), the elevation of the site varies east to west from approximately 390 feet to 380 feet above mean sea level (AMSL).

At the time of the field exploration, the project site was predominantly used for agricultural purposes. In addition, two (2) residential dwellings and seven (7) additional buildings were present at the time of the field exploration. The seven additional buildings included barns, a garage, tack rooms, pump houses, and restrooms. On-site septic systems were reported in the preliminary Environmental Report in association with the existing residential dwellings. Based on visual observations, areas of existing fill generated from the Santa Clara Water District pipeline project mentioned above were not readily identifiable. Also, domestic and irrigation wells were observed on site.

At the time of the field exploration, the southern and southeastern portions of the site were in agricultural use for pepper crops, the western and southwestern portions of the site generally consisted of dried grasses, the northwestern portion of the site consisted of a small vineyard, and the northeastern portion of the site generally consisted of sparse dried grasses and a horse boarding facility. Numerous trees were located along the western portion of the site. Also, several trees were located on the northeastern portion of the site.

3.4 **Anticipated Construction:** Based on the July 28, 2003 preliminary site plan provided to our firm, the proposed construction will include the following structures: 1) nine (9) retail store buildings (8 major tenants and 1 marketplace) will comprise approximately 492,943 square feet in plan dimensions;
2) one (1) store area will comprise approximately 14,000 square feet in plan dimensions; and 3) eight (8) pads will comprise approximately 44,500 square feet in plan dimensions.

The proposed retail development will comprise approximately 56 acres. It is anticipated that the proposed Major Retail Store buildings and other building construction will consist of single story structures with concrete masonry unit, steel, wood, or concrete tilt-up walls, steel and/or wood frame roofs, and concrete slab-on-grade floors. Basements are not anticipated, however, depressed truck ramps and loading docks are anticipated at the retail stores. The project will also include asphaltic concrete and PCC parking and drive areas.

It should be noted that the scope of work requested and provided for this investigation will not meet the geotechnical report requirements for major store tenants, nor does it include evaluation or recommendations for any off-site street improvements. It is anticipated that the column and line loads will vary for each building anticipated within the proposed shopping center. In any case, for purposes of this preliminary geotechnical engineering investigation, maximum column and line loads of 150 kips and 5 kips per lineal foot, respectively, have been assumed for the proposed retail development. It should be noted that once a definitive site plan with specific building types is developed, a design level geotechnical engineering investigation report will be required to address major store requirements, specific design building loads, and building pad earthwork requirements for the different loading and subsurface site conditions.

Since the existing ground surface elevation varies by approximately 10 feet across the site, maximum earthwork cuts and fills of about two (2) to ten (10) feet are expected to achieve level building pads and provide positive site drainage. These depths do not include the depth of over-excavation recommended in this report to provide the recommended depth of engineered fill below foundations. When final grading plans have been prepared, Twining should be provided the opportunity to review the grading plans and provide additional recommendations if necessary.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing program conducted for this investigation are summarized in the following subsections.

4.1 **Field Exploration:** The field exploration consisted of a site reconnaissance, drilling test borings, soil sampling, and standard penetration tests.

4.1.1 **Site Reconnaissance:** The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Mr. Dean Ledgerwood (Twining) on September 1, 2004, Mr. Harry Wise (Twining) and Mr. Barry Annis (Twining) on September 2, 2004. The features noted are described in the background information.

4.1.2 <u>Drilling Test Borings</u>: The depths and locations of test borings were selected based on direction by the client, the size of the proposed structures, type of construction, depth of influence of surface loads, and subsurface soil conditions.

On September 1 and 2, 2004, a total of twelve (12) test borings were drilled in the areas proposed for development to depths ranging from 10 to 51½ feet below site grade (BSG). In addition, six (6) bulk samples of soil were obtained for Resistance (R)-values, expansion index tests, and moisture-density relationship tests. The test boring and bulk sample locations are shown on Drawing No. 2 in Appendix A of this report. Under the direction of a Twining staff geologist, the test borings were drilled using a CME-75 drill rig equipped with 6⅝-inch outside diameter (O.D.) hollow stem augers. The soils encountered in the test borings were logged by the staff geologist. The soil samples collected from the borings were classified in the field in accordance with the Unified Soil Classification System. This classification consisted of particle size, color, and other distinguishing features of the soil.

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of the borings.

Test boring locations were determined by pacing with reference to the existing site features delineated on the preliminary site plan. Elevations of the test borings were not measured as a part of the investigation. The locations of the test borings are described on the boring logs in Appendix B. The test borings were loosely backfilled with material excavated during the drilling operations; thus, some settlement should be anticipated.

4.1.3 <u>Soil Sampling</u>: Standard penetration tests were conducted, and both disturbed and relatively undisturbed soil samples were obtained. The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1⅜-inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by pushing or driving a California modified split barrel ring sampler into the soil. The soil was retained in brass rings, with a 2.5 inch O.D. and 1-inch in height. The lower 6-inch portions of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory.

Soil samples obtained were taken to Twining's laboratory for classification and testing.

4.2 <u>Laboratory Testing</u>: The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and relatively undisturbed samples representative of the subsurface material.

The results of laboratory tests are summarized on Figures Numbers 1 through 30 in Appendix C of this report. These data, along with the field observations, were used to prepare the final test boring logs included in Appendix B of this report.

5.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

5.1 Soil Profile: In two (2) borings (B-3 and B-11) out of the twelve (12) borings drilled, near surface, very stiff, sandy lean clays were encountered from the ground surface to depths of about 3 feet below site grade (BSG). However, in general, the near surface soils encountered at the boring locations consisted of very stiff to hard sandy silts interbedded with medium dense to dense silty sand extending from the ground surface to depths of about 3 feet BSG. From about 3 feet to about 20 feet BSG, the very stiff to hard silts and medium dense to dense silty sands were interbedded with medium dense to very dense silty to clayey gravels. Below a depth of about 20 feet BSG, the interbedded hard silts and medium dense to very dense sands and gravels extended to the maximum depth explored of 51½ feet BSG, with the exception of one (1) boring (B-8) where very stiff to hard sandy clay was encountered between depths of 20 and 25½ feet BSG.

The foregoing is a general summary of the soil conditions encountered in the test borings drilled for this preliminary geotechnical investigation. Detailed descriptions of the soils encountered at each test boring are presented on the logs of borings in Appendix B of this report. The stratification lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual.

5.2 Soils Engineering Properties: The near surface sandy lean clays, encountered in two (2) out of the twelve (12) borings, extending from the ground surface to depths of approximately 3 feet BSG, are very stiff, as indicated by standard penetration resistance, N-values, of 23 blows per foot. The natural moisture contents of the sandy lean clays ranged from 4 to 8 percent. One (1) in-place density test performed on the near surface sandy lean clays indicated a dry density of 103 pounds per cubic foot. The results of two (2) Atterberg Limits tests indicated that the near surface sandy lean clays had liquid limits of 36 and 21,and plasticity indices of 21 and 5, respectively. Two expansion index (EI) tests indicated that the near surface sandy lean clays had very low to low potential for expansion (EI=11 and EI=23).

The near surface sandy silts interbedded with silty sands, encountered in ten (10) of the twelve (12) borings to depths of approximately 3 feet BSG, are medium dense to dense sands and very stiff to hard silts, as indicated by standard penetration resistance, N-values, ranging from 23 to 40 blows per foot and 18 to 54 blows per foot, respectively. The natural moisture contents of the near surface silty sands and sandy silts ranged from 3 to 10 percent. Three (3) in-place density tests performed on the near surface sandy silts interbedded with silty sands indicated dry densities ranging from 108 to 113 pounds per cubic foot. The interbedded soils exhibited high compressibility characteristics as indicated by two consolidation tests (about 13.8 and 14.3 percent consolidation under a load of

8 kips per square foot). Upon inundation, the near surface soils exhibited high apparent collapse potential (about 6.5 and 7.0 percent collapse under a load of 2 kips per square foot). However, upon review of the dry densities, field blow counts, moisture contents and considering the gravel content of the soils, the apparent collapse potential appears to be due to disturbance of the sample caused by driving the sampler. Direct shear tests were conducted on two (2) near surface samples (interbedded silts and sands). The near surface soils had angles of internal friction of 34.3 and 38.2 degrees, and cohesion values of 0 and 95 pounds per square, respectively.

In general, the near surface interbedded soils were underlain by medium dense to very dense gravels interbedded with very stiff to hard sandy silts between the depths of 3 feet and 20 feet BSG as indicated by standard penetration resistance, N-values, ranging from 23 to 73 blows per foot and 18 to 60 blows per foot, respectively. The natural moisture contents of the interbedded gravels and silts ranged from 2 to 11 percent. One (1) in-place density test performed on the interbedded gravels and silts indicated a dry density of 125 pounds per cubic foot.

In general, below a depth of about 20 feet BSG, the interbedded sands and silty soils were underlain by medium dense to very dense gravels interbedded with hard sandy silts and dense to very dense silty sands from about 20 feet to the maximum depth explored of 51½ feet BSG. In one (1) boring (B-8), very stiff to hard sandy clays were encountered between 20 and 25½ feet BSG. The gravels were medium dense to very dense as indicated by standard penetration resistance, N-values, ranging from 27 to 85 blows per foot; the silts were hard as indicated by an N-value of 60 blows per foot; and the sands were dense to very dense as indicated by N-values ranging from 47 to greater than 50 blows per foot. The natural moisture contents of the interbedded gravels, silts, and sands ranged from 1 to 12 percent.

A maximum density/optimum moisture determination test performed on one near-surface silty sand sample collected from the site, indicated a maximum dry density of 125.7 pounds per cubic foot at an optimum moisture content of 9.5 percent.

R-value tests were conducted on two (2) near surface soil samples collected between the depths of ½ and 3 feet BSG. The results of the tests indicate R-values of 17 and 34.

Chemical tests performed on one (1) near surface soil sample indicated a minimum resistivity value of 16,000 ohms/centimeter, a pH value of 5.8, and 0.0008 percent by weight concentrations of chloride and 0.00068 percent by weight concentrations of sulfate.

5.3 **<u>Groundwater Conditions</u>:** Groundwater was not encountered in the test borings drilled at the time of the field exploration (September 1 and 2, 2004). Review of groundwater maps prepared by the Department of Water Resources (DWR) indicates that there are no records of water wells in the nearby Morgan Hill area that would provide information relative to historic groundwater depth. Groundwater was not encountered in any of our borings, with the deepest boring being drilled to 51½ feet. However, the Seismic Hazard Zone Report of the Morgan Hill Quadrangle prepared by the State of California indicates the historic high groundwater depth in the northern portion of the property is 40 feet below the present ground surface.

It should be recognized however, that water table elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

6.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for project design and preparation of geotechnical construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface conditions determined from the investigation and our understanding of the proposed construction.

6.1 Surface Conditions: At the time of the field exploration, the project site was predominantly used for agricultural purposes and included a horse boarding facility. In addition, two (2) residential dwellings and seven (7) additional buildings were present at the time of the field exploration. The seven additional buildings included barns, a garage, tack rooms, pump houses, and restrooms. On-site septic systems are reportedly present on the subject property in association with the residential dwellings. It has been reported by one of the property owners that fill soils were imported to the site as a part of an adjacent pipeline project. However, the extent of fill soils was not discernable upon visual observation of the surface features of the site nor review of a low elevation aerial photograph. Also, domestic and irrigation wells were observed on site.

Vegetation in the southern and southeastern portion of the site generally consisted of pepper crops, the western and southwestern portion of the site generally consisted of dried grasses, the northwestern portion of the site consisted of a small vineyard, and the northeastern portion of the site generally consisted of sparse dried grasses. Numerous trees were located along the western portion of the site, separating the western most parcel from the remainder of the site. Also, several trees were located on the northeastern portion of the site in the area of the horse boarding facilities.

The soils in areas of barns and like facilities, the trees, grasses, etc. will need to be assessed to determine appropriate stripping depths. The soils with excessive organic contents, i.e., greater than 3 percent by dry weight, will need to be stripped from the site in accordance with the recommendations of this report. Additional testing in the form of loss on ignition tests should be performed to determine the minimum stripping depths for the project to reduce costs associated with this activity.

The subsurface structures associated with the existing site improvements (i.e., houses, barns, foundations, wells, septic tanks, leach fields, etc.) will need to be identified and shown on a demolition plan. The resulting excavations will need to be prepared in accordance with the recommendations of this report.

The presence of the undocumented will require that these soils be evaluated to determine their horizontal and vertical extent. These soils will need to be over-excavated and compacted as engineered fill as part of the preparation of the site.

6.2 **Soil Conditions:** The near surface soils exhibited high compressibility and high collapse potential, high shear strength, and poor to fair pavement support characteristics. Also, the near surface sandy clay to sandy silt soils exhibited a very low to low potential for expansion (EI=11 and 23). Based on the compressible nature of the native near surface soils, the near surface soils will not provide adequate support for the proposed improvements in their present condition. Therefore, ground improvement such as over-excavation and compaction will be required to provide adequate support for the proposed structures.

From a geotechnical standpoint, the site is suitable for the proposed construction with regard to support of shallow spread foundations and concrete slabs-on-grade, provided the recommendations contained in this preliminary report are followed. The primary geotechnical concerns are: 1) the presence of undocumented fill soils; 2) the presence of disturbed, highly compressible and collapsible, native, near surface soils encountered from the ground surface extending to about 3 feet BSG; 3) the potential to incorporate over-sized material (gravel and cobbles) within the footing zone; and 4) the potential to encounter over-sized materials (gravel and cobbles) during grading operations.

One of the primary geotechnical concerns is the potential to encounter over-sized material (gravel and cobbles) within the planned excavation depths. If these over sized materials (greater than 3 inches along the largest dimension) are encountered during the over-excavation for the building pads, additional excavating equipment may be necessary to excavate the soils with gravels and cobbles. It also may be difficult to break up large fragments for compaction efforts of over-excavated and scarified areas. If the cobbles cannot be broken up into fragments that are three (3) inches or less in size along the largest dimension, the contractor should remove the over sized fragments from the fill material by screening or other techniques approved by Twining. The screening activity would be anticipated to increase the cost of the earthwork. Neatly cut footings will be difficult to obtain due to the presence of oversized materials unless materials placed in the foundation zone are screened to remove oversize materials. In order to better determine the requirements for screening, it is recommended that test pits be excavated at this site under the direction of Twining. Potential grading, excavating and underground contractors should be present onsite to make their own observations of the gravels and cobbles in the soils exposed in the test pits. This activity will assist the contractors in preparation of bids. It has been reported by one of the property owners that fill soils were imported to the site as a part of an adjacent pipeline project. However, the extent of fill soils was not discernable upon visual observation of the surface features of the site nor review of a low elevation aerial photo. Therefore, as part of the backhoe exploration, the extent of onsite fill soils should be further evaluated.

6.3 **Site Preparation:** Over-excavation and placement of engineered fill below foundations are recommended to limit static settlements to 1 inch total and ½ inch in 40 feet differential. Site preparation should include stripping and removal of the existing structures and associated improvements, trees, grasses, organic debris, as well as over-excavation and placement of engineered fill below foundations, slabs and flatwork areas. Stripping should be conducted prior to over-excavation. The proper removal of trees and their associated root structures is an important aspect of this project and should be properly planned and monitored. A demolition plan should be prepared for the project.

An implementation plan for demolition should be developed by the contractor and should include surveying the site. The plan should specify how the contractor proposes to remove the building structures, septic tanks and leach lines, wells and irrigation lines, and other underground utilities, trees, roots, and organic matter generated during the removal process, and how the excavations and loose soils generated during this process will be addressed.

If large roots are allowed to remain in the soils and organically decay over time, voids could develop and/or an organic stench of decaying vegetation could permeate overlying structures, pavements, and exterior slabs.

In addition to removing structures on site (i.e. two residences, barns, sheds, garages), the contractor should also locate and remove foundations, septic tanks and leach lines, including gravel, to at least 1 foot below the bottom of the subsurface feature to be removed. Site preparation should also include removal or relocation of any irrigation systems present at the site. Irrigation pipe should be completely removed and not crushed in-place and buried.

After stripping and removal of surface and subsurface structures, over-excavation is recommended to remove disturbed soils and reduce the potential for static settlement of the new structures. The overbuild zone should extend to a minimum of 10 feet beyond the building perimeters. This area should be over-excavated to the same depth that is required for over-excavation underneath the footings, adjacent flatwork, exterior columns, and canopies or overhangs.

Over-excavation should also be conducted in pavement and exterior flatwork (e.g., parking areas, drive areas and sidewalks) to over-excavate compressible soils and establish a firm, compacted subgrade as recommended herein. The zone of over-excavation and compaction should extend laterally a minimum of 3 feet outside the perimeters of the proposed flatwork and pavements.

Water wells were noted on site. The water wells (if not to be preserved or used after construction) should be abandoned in accordance with state and local requirements as well as the recommendations contained in this report.

6.4 **Foundations:** Considering the nature of the near surface soils encountered at the boring locations, the potential for excessive settlement of proposed foundations and slabs is considered moderate. Settlement estimates were made based on a bearing pressure of 2,500 pounds per square foot. Twining should be retained to provide a supplemental evaluation and recommendations when the building loads become available. All foundations for the project should be supported on at least 24 inches of engineered fill in order to provide a uniform bearing surface for the foundations and slabs, and limit total and differential static settlements to 1-inch and ½-inch in 40 feet, respectively.

The maximum allowable soil bearing pressure of 2,500 pounds per square foot was selected to satisfy both the settlement criteria and Terzaghi bearing capacity equations for spread foundations. A factor of safety of 3 was used to determine the allowable bearing capacity based on Terzaghi equations. Schmertmann's method was used to estimate foundation settlements.

6.5 **Interior Slabs-on-Grade:** Interior floor slabs should be supported on at least six (6) inches of Class 2 aggregate base (AB) over 12 inches of granular non-expansive engineered over engineered fill extending to the minimum depth of fill recommended under foundations indicated in this report. The Class 2 AB material is recommended for structural purposes, to provide a capillary break, and to provide a working surface during construction.

Several issues need to be considered to avoid damaging slabs during construction. These issues include: 1) using perimeter pour-strips at tilt-wall locations (if used) to avoid damage to slab-wall connections if tilt-up construction is used; 2) differential slab movement at interior columns; 3) aggregate base sections below the slabs; and 4) crane and heavy equipment loads on the slabs.

For tilt wall construction, depending on the sequence of slab loading and the location of wall panel casting, damage to slabs from differential loading conditions could occur. It has been our experience that a concentrated amount of differential movement and damage at the slab-to-perimeter footing location can occur as wall panels are placed and the footing is loaded. This typical procedure results in cracking of slabs and foundations due to differential movement. This potential damage can be reduced by leaving a perimeter pour strip between the wall footing and the adjacent slabs. After the walls are erected and a majority of the differential movement has occurred; the pour strip can be placed.

Often interior column construction can damage the overlying slabs. In some cases, the subgrade preparation for the slab is not continuous across the top of spread footings. Often the zone above the top of structural footings is backfilled with concrete during slab placement. This results in a differential slab support condition which often causes cracking at the soil/base-to-concrete transition. This crack appears as an outline of the underlying footing at the floor surface. The potential for this type of slab cracking can be reduced by backfilling the zone above the top of the footing with an approved backfill material and/or an aggregate base section below the floor slab. This procedure will provide more uniform support for the slabs which should reduce the potential for cracking.

It has been our experience that placing concrete for the concrete slabs by the tailgating method can cause subgrade instability due to the high frequency of concrete trucks which travel across the prepared subgrade. Compacted subgrade can experience instability under high traffic loads resulting in heaving and depressions in the subgrade during critical pours. This condition becomes more critical during wet winter and spring months. Six (6) inches of aggregate base can reduce the potential for instability under the high frequency loading of concrete trucks. Also, the improved support characteristics of the aggregate base can be used in the design of the slab sections. Therefore, it is recommended to utilize a slab design with at least 6 inches of aggregate base for constructability and design purposes.

Finally, it is expected that erection of concrete tilt-up wall panels and roof steel will require cranes and heavy equipment. It should be noted that larger cranes impart intense loads on slabs and pavements. The loaded track pressure of any crane which will operate on slabs or pavements should be considered in the design.

Fine grained native and engineered fill soils may become unstable during grading; and therefore, could require stabilization. Stabilization may include placing a geotextile fabric and aggregate base materials, and/or chemical treatment, or a combination of these to stabilize soils. For bidding purposes, costs for chemical treatment, including 5 percent by weight high calcium quick lime should be provided as a bid alternate.

Interior floor slabs should be supported on 6 inches of Class 2 aggregate base (AB) over 12 inches of non-expansive engineered fill. The Class 2 AB material is recommended for structural purposes, to provide a capillary break, and to provide a working surface during construction.

6.6 <u>Ground Rupture and Seismic Ground Motion</u>: The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest presently known active or potentially active fault is the Calaveras Fault (South of the Calavaras Reservoir segment), located about 3 miles (4¾ km) east of the site. Therefore, the potential for fault rupture at the site is considered low.

Seismic ground motion parameters were developed for use in the liquefaction hazard analyses. The "design basis ground motion," Section 1627 of the California Building Code (CBC), is defined as the seismic ground motion having a 10 percent probability of being exceeded in a 50-year period. The probabilistic analyses described in this section was used to determine the design basis ground motion.

Probabilistic ground motion evaluation requires use of a seismicity model and ground motion attenuation functions to approximate the modification of seismic waves between the earthquake hypocenter (source) and the site. The seismicity model, including the fault locations and fault parameters (such as slip rate, fault length, magnitude and rupture area) of faults capable of impacting the site, was based on published geologic papers and corresponds with those listed in the California Geological Survey (CGS) database entitled "California Fault Parameters." Multiple probabilistic evaluations were conducted using the FRISKSP computer program and the faults indicated as those active and potentially active faults listed in the "California Fault Parameters" database.

As an initial step, our evaluation utilized the Boore (1997) attenuation relationship and included faults located within approximately 100 kilometers of the site. The resultant peak horizontal ground acceleration with a 10% probability of being exceeded in 50 years (the design basis ground motion) was 0.83g. The next step of our evaluation considered the average of the predicted design basis ground motions for four separate analyses incorporating four ground motion attenuation relationships including Boore (1997), Sadigh (1997), Idriss (1994), and Abrahamson and Silva (1997) and the faults within 100 kilometers of the site. The average of the design basis site accelerations based on the above attenuation relationships was determined to be 0.89g. Accordingly, a ground motion of 0.89g was selected for use in the liquefaction analyses. This represents a value not weighted for magnitude. Magnitude weighting is conducted in the liquefaction analysis.

Hazard deaggregation was conducted using the FRISKSP computer program. The results indicate that an earthquake magnitude of 7.9 represents the predominant earthquake magnitude for the site. This earthquake magnitude was used with the above ground motion estimate for the liquefaction analyses.

It is expected that the 2001 CBC will be used for structural design, and that seismic site coefficients are needed for design. Based on the CBC, the site classification is estimated to be a stiff soil S_D site with standard penetration resistance N-values averaging between 15 and 50 blows per foot in the upper 100 feet BSG.

The site coefficients for acceleration and velocity are based on the distance and activity of the local faults. Digitized seismic models published by the CGS indicate that two active faults are located within 15 kilometers of the site. These faults are the South of the Calaveras reservoir segment of the Calaveras Fault (distance about 4.8 km, M_m=6.2, slip rate 15.0 mm/year) and the Sargent Fault (distance 12.2 km, M_m=6.8, slip rate 3.0 mm/year). Based on the magnitudes and slip rates, these faults are classified as Source Type B faults by the CBC.

A table providing the recommended seismic coefficients for the project site is included in the Foundation Recommendations Section of this report.

6.7 <u>Liquefaction and Seismic Settlement</u>: Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of excessive pore water pressure induced by shearing strains. Lateral and vertical movement of the soil mass, combined with loss of bearing usually results in the liquified zone. Research has shown that liquefaction potential of soil deposits induced by earthquake activity depends on soil types, void ratio, groundwater conditions, duration of shaking, and confining pressure over the potentially liquefiable soil mass. Fine, well sorted, loose sand, high groundwater conditions, higher intensity earthquakes, and relatively long duration of ground shaking are the requisite conditions for liquefaction. It should be noted that the northern portion of the site (approximately ¼ of the site) is located in a liquefaction hazard zone delineated in response to the Seismic Hazards Mapping Act. Therefore, a preliminary liquefaction/seismic settlement analyses was conducted as a part of this report.

Liquefaction and seismic settlement analyses were conducted based on the soils encountered in the test borings and the results of laboratory testing. The analyses were conducted based on the computer program LIQUEFY2 developed by Mr. Thomas F. Blake. A design basis earthquake horizontal ground acceleration of 0.89g and a design earthquake magnitude of 7.9 were used in the analysis. The N-values generated based on the SPT results were used to determine the cyclic stress ratio needed to initiate liquefaction. Soil parameters, such as wet unit weight, N-value, fines content, and depth of N-value tests, were input for the soils layers encountered throughout the depths explored (see test boring logs, Appendix B).

One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils, however, seismic settlements are typically largest where liquefaction occurs (i.e., in saturated soils). Estimates of total and differential seismic settlements on the order of 0.25 inches and 0.25 inches in 40 feet, respectively, were predicted. In addition, static total and differential settlements on the order of 1 inch and ½ inch in 40 feet, respectively, are anticipated at the subject site. Therefore, combined settlements (static and seismic) of 1.25 inches total and 0.75 inches in 40 feet differential should be anticipated for design.

Based on the depth to groundwater below the site (not encountered in test borings drilled to 50 feet), the potential for impact to the site from liquefaction effects due to loss of foundation bearing capacity, surface manifestations, and lateral spreading is considered negligible. Considering the shallow depth to dense to very dense gravelly soils, a total seismic settlement on the order of 1/4-inch would be expected to occur under shaking from the design-basis earthquake (0.89g and a magnitude of 7.9).

However, it should be noted that the northern portion of the site (approximately ¼ of the site) is located within an area of potential liquefaction as identified by the CGS on the State of California Seismic Hazard Zones, Morgan Hill Quadrangle, dated April 17, 2004, available on their website at (http://gmw.consrv.ca.gov/shmp/download/pdf/pzn_morgh.pdf). The map shows areas where historical occurrence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693©) would be required. However, based on the results of the preliminary evaluation included herein, the risk of liquefaction at this site is considered low. Due to the potential variability of the subsurface soils and depth to groundwater across the site, it is recommended that the proposed structures be evaluated on a case by case basis as a part of future design level geotechnical engineering investigations.

6.8 Asphaltic Concrete Pavements: Preliminary asphaltic concrete pavement structural sections are presented in the recommendations section of this report. The structural sections were designed using the gravel equivalent method in accordance with Chapter 600 of the California Department of Transportation Highways Design Manual (fourth edition). The structural vehicle loadings were based on typical traffic loadings for this type and size of facility. The analyses were based on a range of traffic index values from 5.0 to 8.0. Traffic indices for the project should be selected by the project Civil Engineer. If the pavements are placed prior to construction, the additional construction traffic should be considered in the selection of the design traffic index. If more frequent truck traffic is anticipated than that indicated by the maximum traffic index, Twining should be contacted to re-evaluate the traffic index values.

The anticipated subgrade soils are silty sands and sandy silts. The subgrade support characteristics of the native soils were evaluated by Resistance (R)-value tests. The results of two tests, obtained within the upper 3 feet BSG, indicated the soils had R-values of 17 and 32. For the purpose of design, an R-value of 17 was used.

6.9 **Portland Cement Concrete (PCC) Pavements:** Recommendations for Portland cement concrete pavement structural sections are presented in this report. The structural section was determined based primarily on the methods detailed in the Portland Cement Association "Thickness Design of Highway and Street Pavements." PCC sections for exterior or interior hard rubber or steel wheel forklifts were not considered in the scope of this investigation.

The anticipated range of traffic selected for the PCC pavement sections ranged from 3 trucks per day to 30 trucks per day (all vehicle counts are one direction).

The PCC pavement sections were designed for a life of 20 years, a load safety factor of 1.1, a single axle weight of 16,000 pounds, a tandem axle weight of 36,000 pounds, and a modulus of rupture of 550 pounds per square inch (compressive strength of 4,000 psi) at 28 days for concrete. The design R-value resulted in a k-value of 80 psi/in. A higher k-value than the subgrade k-value is provided for this pavement section, based on a 6-inch layer of Class 2 aggregate base material (minimum R-value of 78) recommended below PCC slabs. Therefore, a k-value of 150 psi/in at the top of the aggregate base was used in design, and can be used for slab-on-grade design for interior floors subject to vehicle loads which are underlain by 6-inches of AB.

6.10 **Corrosion Protection:** The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. The rate of deterioration depends on soil resistivity, texture, acidity, and chemical concentration. The evaluation was based on the results of an analysis of one (1) near-surface soil sample collected from the surface to depths of about 5 feet BSG. The chemical analyses results indicate a resistivity value of 16,000 ohms/centimeter and a pH value of 5.8. Based on the resistivity value, the soils exhibit "mildly corrosive" corrosion potential. In addition, the results of soil sample analyses indicate 0.00068 percent by dry weight concentrations of sulfate and 0.0008 percent by dry weigh concentrations of chloride. If piping or concrete are placed in contact with deeper soils or imported engineered fills, these soils should be analyzed to evaluate their corrosion potential.

7.0 CONCLUSIONS

Based on the subsurface soils data collected during the field and laboratory investigations, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, we present the following general conclusions.

7.1 The site is considered suitable for the proposed construction with regard to support of foundations and concrete slabs-on-grade, provided the recommendations contained in this report are followed. It should be noted that the recommended design consultation and construction monitoring by Twining are integral to this conclusion.

7.2 In two (2) borings (B-3 and B-11) out of the twelve (12) borings drilled, near surface, very stiff, sandy lean clays were encountered from the ground surface extending to depths of about 3 feet below site grade (BSG). However, in general, the near surface soils encountered at the boring locations consisted of very stiff to hard sandy silts interbedded with medium dense to dense silty sand extending from the ground surface to a depth of about 3 feet BSG. From about 3 feet to about 20 feet BSG, the very stiff to hard silts and medium dense to dense silty sands were interbedded with medium dense to very dense silty to clayey gravels. Below a depth of about 20 feet BSG, the interbedded hard silts and medium dense to very dense sands and gravels extended to the maximum depth explored of 51½ feet BSG, with the exception of one (1) boring (B-8) where very stiff to hard sandy clay was encountered between 20 and 25½ feet BSG.

7.3 The near surface soils exhibited high compressibility and high collapse potential, high shear strength, and poor to fair pavement support characteristics. Near surface sandy lean clay soils were encountered in two (2) of the twelve (12) borings. Atterberg Limits tests conducted on two soil samples indicated that the near surface sandy lean clays had liquid limits of 36 and 21,and plasticity indices of 21 and 5, respectively. Also, the near surface sandy clay to sandy silt soils exhibited a very low to low potential for expansion (EI=11 and 23).

7.4 One of the primary geotechnical concerns is the potential to encounter over-sized material (gravel and cobbles) within the planned excavation depths. If these over-sized materials (greater than 3 inches along the largest dimension) are encountered during the over-excavation for the building pads, additional excavating equipment may be necessary to excavate the soils with gravels and cobbles. It also may be difficult to break up large fragments for compaction efforts of over-excavated and scarified areas. If the cobbles cannot be broken up into fragments that are three (3) inches or less in size along the largest dimension, the contractor should remove the over-sized fragments from the fill material by screening or other techniques approved by Twining. The screening activity would be anticipated to increase the costs of the earthwork. Neatly cut footings will be difficult to obtain due to the presence of over-sized materials. In order to better determine the requirements for screening, it is recommended that test pits be excavated at this site under the direction of Twining. Potential grading, excavating and underground contractors should be present onsite to make their own observations of the gravels and cobbles in the soils exposed in the test pits. This activity will assist the contractors in preparation of bids. It has been reported by one of the property owners that fill soils were imported to the site as a part of an adjacent pipeline project. However, the extent of fill soils was not discernable upon visual observation of the surface features of the site nor review of a low elevation aerial photograph. Therefore, as part of the test pit exploration, the extent of onsite fill soils can be further evaluated.

7.5 To reduce the potential for static total and differential settlements of the foundations, shallow spread footings placed entirely on at least 24 inches of engineered fill, engineered fill which extends to at least 36 inches below preconstruction site grades, or engineered fill which extends to at least 12 inches below improvements to be removed, whichever is deeper, should reduce the potential total and differential static settlements to 1-inch and ½-inch in 40 feet, respectively.

7.6 One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils, however, seismic settlements are typically largest where liquefaction occurs (i.e., in saturated soils). Estimates of total and differential seismic settlement on the order of 0.25 inches and 0.25 inches in 40 feet, respectively, were predicted. In addition, static total and differential settlements on the order of 1 inch and ½ inch, respectively, are anticipated at the subject site. Therefore, combined settlements (static and seismic) of 1.25 inches total and 0.75 inches in 40 feet differential should be anticipated for design.

7.7 The results of a soil sample analysis indicate that the near-surface soils exhibit a "mildly corrosive" corrosion potential to buried metal objects.

7.8 The results of soil sample analyses indicate sulfate concentrations of 0.00068 by percent weight. Therefore, a low potential for sulfate attack on concrete placed in contact with near-surface soils is anticipated.

7.9 The near-surface soils exhibit poor to fair characteristics for pavements. The subgrade support characteristics of the native soils were evaluated by Resistance (R)-value tests. The results of two tests, obtained within the upper 3 feet BSG, indicated the soils had R-values of 17 and 32. For the purpose of design, an R-value of 17 was used. Pavement section recommendations for various traffic indexes are included in the recommendations section of this report.

7.10 Groundwater was not encountered in the test borings drilled to a maximum depth of 51½ feet BSG. However, the State of California Seismic Hazard Zone report indicates historic high groundwater below the northern portion of the site is 40 feet. It should be recognized however, that water table elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors.

7.11 The northern portion of the site (approximately ¼ of the site) is identified by the State of California as being in an area of potential risk of liquefaction. The southern portion of the site is not located within a liquefaction hazard area identified by the State of California considering that the historic groundwater depth is greater than 50 feet. Based on the results of the preliminary evaluation included herein, the risk of liquefaction impacts at this site are considered low. It is recommended that the proposed structures be evaluated on a case by case basis as a part of future design level geotechnical engineering investigations.

7.12 The site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest mapped active fault is located about 3 miles east of the site. Therefore, the potential for fault rupture at the site is considered low.

8.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project site, we present the following preliminary recommendations for use in preliminary project design and construction. However, this report should be considered in its entirety as a preliminary geotechnical engineering investigation. Additional design level geotechnical engineering investigation reports will be required in the future. When applying the preliminary recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered. The recommended design consultation and construction monitoring by Twining are integral to the proper application of the preliminary recommendations.

8.1 General

8.1.1 Grading and structural plans were not available at the time this report was prepared. Thus, an assumption was made that the proposed finished grade would not change from the existing site grade by more than about 5 feet. If the finished grade is higher or lower, the recommendations presented in this report may not be appropriate for the changed conditions. Twining should be provided the opportunity to review the grading plans and foundation plans before the plans are released for bidding purposes so that any relevant recommendations can be presented.

8.1.2 A preconstruction meeting including, as a minimum, the owner, general contractor, foundation and paving subcontractors, civil engineer, and Twining should be scheduled at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling.

8.1.3 Contractors should be aware that areas proposed for pavements and slabs-on-grade adjacent to the proposed building and/or within the overbuild zone should incorporate the more stringent requirements for aggregate base, non-expansive soils and native soil moisture conditioning recommended in this report for interior slabs-on-grade, and AC pavements.

8.1.4 If any city, county, and/or state standards are cited on the plans or specifications, these standards should be followed in addition to the recommendations in this report.

8.1.5 A demolition plan should be developed by the contractor and should include a survey of the site. The plan should specify how the contractor proposes to remove the building structures, septic tanks and leach lines, irrigation lines and other underground utilities, trees, roots, and organic matter generated during the removal process and how the excavations and loose soils generated during this process will be addressed. Demolition of the existing structures present at the site should be observed and monitored by a representative of Twining in order to verify and document complete removal of any subsurface structures or foundations.

8.2 Site Grading and Drainage

8.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades which are not covered by pavements or walkways should be sloped a minimum of two (2) percent for a distance of at least five (5) feet away from the structure to preclude ponding of water adjacent to foundations. Adjacent exterior grades which are paved should be sloped at least one (1) percent away from the foundations.

8.2.2 Surface water must not be allowed to pond adjacent to the building foundations. To preclude this, it is recommended to provide rain gutters and direct all water from roof drains into closed conduits that are connected to an acceptable discharge area away from the building foundations, or directly into the site storm drain system. Excessive irrigation must be avoided. Minimal irrigation such as low volume sprinklers are highly recommended.

8.2.3 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). It is recommended to use plants with low water requirements.

8.2.4 Perimeter curbs should be extended into the subgrade (below the base section) where irrigated landscape areas meet pavements or other measures taken to reduce the potential for moisture from migrating into the aggregate base sections.

8.2.5 It is not recommended to place landscape or planted areas adjacent to the structure. Trees should be setback from the proposed structure at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from the proposed building.

8.3 Site Preparation

8.3.1 All topsoil, trees, vegetation, organic material, irrigation lines, water lines, water wells, sump pumps, and debris should be removed from the proposed building and pavement areas. The general depth of stripping should be sufficiently deep to remove the root systems and organic material. For estimating purposes, a minimum stripping depth of 6 inches should be used. The actual depth of stripping should be reviewed by Twining at the time of construction. It is possible that deeper stripping may be required if any roots larger than ¼-inch are encountered during grading and in localized areas, such as low areas where water may pond. Stripping should extend laterally a minimum of 10 feet outside the building and pavement perimeters. These materials will not be suitable for use as engineered fill; however, stripped topsoil may be stockpiled and reused in landscape areas at the discretion of the owner. All underground utility lines, pipes and other structures associated with the existing improvements (not to be used) should be removed. The future disposition of existing utilities on the site are currently unknown. If these utilities are to be abandoned, it is recommended to remove these utilities and the associated backfill unless documentation is provided that the backfill was placed as engineered fill. If these utilities are to remain in service, the backfill should be tested to determine if the existing backfill is properly compacted and capable of supporting the improvement proposed to be constructed in these areas. These areas should be designated on the project civil drawings.

8.3.2 Stripping should be observed by Twining. Roots larger than ¼-inch, and any accumulation of roots that result in an organic content greater than 3 percent by weight as determined by loss-on-ignition tests, should be removed. The exposed subgrade in the excavations should be scarified and compacted as engineered fill to a depth of 8 inches and the excavation backfilled with engineered fill. It is recommended that additional testing be performed during the design level studies to better define the depth of stripping required for this project.

8.3.3 All existing structures, foundations, floor slabs, underground utilities, leach lines, septic tanks, wells, pits, etc. should be shown on a demolition plan, located during construction, and entirely removed. Also, any other subsurface structure or accumulation of organic material not identified on the demolition plan but encountered during construction should be removed. The resulting excavations, which should extend to at least 12 inches below the structure to be removed, should be cleaned of all loose or organic material to exposed native soils, then scarified to a depth of 8 inches, conditioned, and compacted as engineered fill and the excavation backfilled with engineered fill.

8.3.4 As a minimum, the existing features to be removed include (but are not limited to) the existing residential structures, storage structures, associated sewage disposal systems, leach lines, wells, irrigation lines, and water wells. All foundations, floor slabs, fence posts, underground utilities, septic tanks, leach lines, etc. should be located and entirely removed. Septic systems and leach lines should be removed in accordance with state and county regulations. The resulting excavations, which should extend to at least 12 inches below the structure to be removed, should be cleaned of all loose or organic material to exposed native soils, then scarified to a depth of 8 inches, conditioned, and compacted as engineered fill and the excavation backfilled with engineered fill.

8.3.5 Wells were observed during the field investigation. All wells scheduled for demolition should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner and Twining upon completion. At a minimum, wells in building areas (and within 5 feet of building perimeters), should have their casings removed to a depth of at least 10 feet below site grade or finished pad grade. In parking lot or landscape areas, the casings should be removed to a depth of at least 7 feet below site grade or finished grade. The wells should be capped with concrete and the resulting excavations should be backfilled as engineered fill. If the wells are related to an environmental investigation regulated by a local, state, or federal agency, the removal of these wells should be performed under the oversight of the regulatory agency.

8.3.6 After stripping, removal of structures, and removal of loose soils, the building pad areas should be over-excavated so that the foundations will be supported on engineered fill to reduce settlement. The minimum depths of engineered fill recommended are provided in subsection 8.5. The zone of over-excavation should include all the building interiors and extend laterally a minimum of ten (10) feet beyond the buildings, vestibules, utility pads, and sidewalks, stairs, ramps, etc. Any soft or unstable areas identified during compaction of the bottom of the over-excavation should be removed and compacted as engineered fill.

8.3.7 Contractors should be aware that areas proposed for pavements and slabs-on-grade adjacent to the proposed building and/or within the overbuild zone should incorporate the more stringent requirements for non-expansive soils and native soil moisture conditioning recommended in this report for interior slabs-on-grade, AC pavements, and PCC pavements.

8.3.8 It is recommended that pavement sections and exterior flatwork outside the over-build zone be underlain by at least 12 inches of non-expansive engineered fill, engineered fill extending to at least 18 inches below preconstruction site grades, or engineered fill extending to at least 12 inches below improvements to be removed, whichever provides the deeper fill. Prior to placement of fill, proof rolling, under the observation of Twining, should be performed. The zone of over-excavation should include all the pavement areas and extend laterally a minimum of three (3) feet beyond the edge of pavements or curbs. Exterior flatwork should be underlain by at least four (4) inches of aggregate base over eight (8) inches of non-expansive engineered fill directly below the concrete.

8.3.9 It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Twining is not responsible for measuring and verifying the horizontal and vertical extent of over-excavation and compaction. The contractor should verify in writing to the owner and to Twining that the horizontal and vertical over-excavation limits were completed in conformance with the recommendations of this report, the project plans, and the specifications (the most stringent applies). It is recommended that this verification be performed by a licensed surveyor. This verification should be provided prior to requesting pad certification from Twining or excavating for foundations.

8.3.10 All fill required to bring the site to final grade should be placed as engineered fill. In addition, all native soils over-excavated from the site should be moisture conditioned and compacted as engineered fill.

8.3.11 It is anticipated that some subgrade instability may occur if earthwork operations are conducted in wet weather or if the in-situ soils are over optimum moisture content. The degree of instability will depend on the actual moisture content of the soils at the time of construction. Fine grained native and engineered fill soils may become unstable during grading; and therefore, could require stabilization. Stabilization may include placing a geotextile fabric and aggregate base materials, and/or chemical treatment, or a combination of these to stabilize soils. For bidding purposes, costs for chemical treatment, including 5 percent by weight high calcium quick lime should be provided as a bid alternative. The actual method employed to stabilize the bottom of the excavation or pavement subgrade should be selected at the time of construction. In addition, Twining should evaluate individual subgrade instability situations and provide specific recommendations on an as needed basis.

8.4 Engineered Fill

8.4.1 The on-site soils encountered are predominantly sandy silts and gravels. The silt soils will be suitable for use as fill material to support the structural loads, provided they are free of oversized particles greater than 3 inches, organic materials and debris and moisture conditioned to between optimum moisture content and three (3) percent above the optimum moisture content at the time of placement. These soils should not be used within 12 inches of the bottom of slabs-on-grade. The soils in the upper 12 inches should be imported granular soils and/or aggregate base. If soils other than those considered in this report are encountered, Twining should be notified to provide alternate recommendations.

8.4.2 The compactibility of the native soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, we recommend that they be evaluated by the contractor during preparation of bids and construction of the project.

8.4.3 Import fill soil should be nonexpansive and granular in nature with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	50 - 100
Percent Passing No. 200 Sieve	10 - 30
Plasticity Index	Less than 10
Expansion Index (UBC 29-2)	Less than 10
Organics	3% maximum by dry weight
R-Value	Minimum 30
Sulfates	< 0.05 % by weight
Minimum Resistivity	> 5,000 ohm/cm

Prior to being transported to the site, the import fill material should be tested and approved by Twining. Prior to being transported to the site, the import material shall be approved by Twining; and certified by the contractor or suppliers (to the satisfaction of the owner) that the soils do not contain any environmental contaminants. Documentation should be provided to Twining as well as to the client that the imported soils are free of any substance regulated by local, state, or federal agency, or any contaminant which may adversely affect the proposed development.

8.4.4 Recycled materials (such as asphaltic concrete or Portland cement concrete) should not be used within 5 feet of any improvement without approval by the owner and Twining. Contractors should not assume that recycled materials can be used in preparing bids for the project without approval by the owner and Twining.

8.4.5 Engineered fill soil should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between optimum moisture content and three (3) percent above the optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. For fills placed deeper than 5 feet below finished grades, these soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned as required, and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.4.6 Any open graded gravel or rock material such as ¾-inch crushed rock or ½-inch crushed rock used as backfill should be placed in 6-inch lifts and compacted using a vibratory compactor to a non-yielding condition as determined by Twining. Each lift must be approved prior by Twining to placing the next lift. All open graded materials should be encased in a geotextile filter fabric to prevent migration of fine grained soils into the porous material. The contractor shall obtain a certification from the fabric manufacturer that the fabric is suitable for the intended use.

8.5 Foundations

8.5.1 It is assumed that the maximum column and continuous footing loads for the development will be 150 kips and 5 kips per lineal foot, respectively. Structural loads for the proposed structures may be supported on spread or continuous footings placed entirely on at least 24 inches of engineered fill, engineered fill that extends to at least 36 inches below preconstruction site grades, or engineered fill that extends to at least 12 inches below improvements to be removed and undocumented fill, whichever provides the deeper fill. The building pad areas, including the over-build zone extending 10 feet beyond the building perimeter, should be over-excavated to the same depth that is required for over-excavation beneath the bottom of the footings. Spread and continuous footings may be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.5.2 One of the primary geotechnical concerns is the potential to encounter oversized, greater than 3 inch, gravel and cobble material from the surface to depths of about 5 feet BSG. If these larger materials are encountered during the over-excavation for the building pads, they should be broken up/crushed into fragments that are three (3) inches or less in size along the largest dimension. If the cobbles cannot be broken up into fragments that are three (3) inches or less in size along the largest dimension, the contractor should remove the over sized fragments from the fill material by screening or other techniques approved by Twining. Neatly cut footings will be difficult to obtain due to the presence of oversized materials. In order to better determine the requirements for prescreening, it is recommended that test pits be excavated at this site under the direction of Twining. Potential grading, excavating and underground contractors should be present onsite to make their own observations of the gravels and cobbles in the soils exposed in the test pits. This activity will assist the contractors in preparation of bids. It has been reported by one of the property owners that fill soils were hauled in to the site as a part of an adjacent pipeline project. However, the extent of fill soils was not discernable upon visual observation of the surface features of the site nor review of a low elevation aerial photo. Therefore, as part of the backhoe exploration, the limits of onsite fill soils should be further evaluated.

8.5.3 The footings should have a minimum depth of 18 inches below rough pad grade (note: 24 inches for major stores or two-story buildings) or below the lowest adjacent exterior grade, whichever is lower. Footings should have a minimum width of 12 inches (note: 15 inches for major stores or two-story buildings), regardless of load. Perimeter footings should be continuous around the entire building perimeters.

8.5.4 Total and differential static settlements of 1-inch and ½-inch, respectively, over a horizontal distance of 40 feet should be anticipated for design. Combined static and seismic total and differential settlements of 1 1/4 inch and 3/4 inch, respectively, over a horizontal distance of 40 feet should be anticipated for design

8.5.5 The assumed building loads cited in this report should be confirmed by the structural engineer. If the assumed loads are different than the actual design loads, Twining should be contacted to provide supplemental recommendations. It should be noted that design level geotechnical reports will be required for all the buildings.

8.5.6 Twining should observe the bottoms and sides of the foundation excavations to verify that the excavations are properly moisture conditioned, and comply with the recommendations of the final geotechnical report prior to placement of plumping, reinforcement, and concrete. If loose soils or soils with moisture contents below optimum are encountered, the contractor should request written recommendations from Twining to address the loose soils and moisture conditions the foundation excavations.

8.5.7 The foundations should be designed and reinforced for the anticipated differential settlements. A structural engineer experienced in foundation design should recommend the reinforcement, thickness, design details and concrete specifications for the foundations based on: 1) a total settlement of 1-inch, 2) a differential settlement of ½-inch in 40 linear feet of continuous footings; 3) a differential settlement of ½-inch between isolated column footings; and) combined static and seismic total and differential settlements of 1 1/4 inch and 3/4 inch, respectively over a horizontal distance of 40 feet.

8.5.8 Structural loads for miscellaneous foundations (such as retaining walls, sound walls, screen walls, pylon signs, monument signs, etc.) should be evaluated on a case by case basis to develop supplemental recommendations for site preparation and foundation design. In lieu of a case by case evaluation, miscellaneous foundations may be supported on spread or continuous footings placed entirely on at least 24nches of engineered fill, engineered fill to a depth of at least 36 inches below preconstruction site grade, or at least 12 inches below improvements to be removed, whichever is deeper. Subgrade preparation should be conducted below the structures and extend throughout the over-build zone as described in this report. The resulting excavations should be cleaned of all loose or organic material, and the exposed native soils should be scarified to a depth of 8 inches, moisture conditioned, compacted as engineered fill, and the excavation then backfilled with engineered fill. Spread and continuous footings may be designed for a maximum allowable soil bearing pressure of 2,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.5.9 The following factors were developed based on the tables in Chapter 16 of the 2001 California Building Code (CBC) and the digitized active fault locations published by CGS.

Seismic Factor	UBC Value
Soil Type	S_D
Source Types	B
Near Source Acceleration Factor, Na	1.0
Near Source Velocity Factor, Nv	1.2
Seismic Acceleration Coefficient, Ca	0.45
Seismic Velocity Coefficient, Cv	0.78

8.5.10 If soft or unstable soils are encountered during excavation or compaction operations, proof rolling, under the observation of Twining, should be performed to examine the soil conditions and provided additional recommendations, as needed.

8.6 Frictional Coefficient and Earth Pressures

8.6.1 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads (areas of slabs underlain by a synthetic moisture barrier cannot be considered). An ultimate coefficient of friction of 0.50, reduced by an appropriate factor of safety, can be used for design.

8.6.2 The ultimate passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 265 pounds per cubic foot. This value should be reduced by an appropriate factor of safety.

8.6.3 The passive pressure was calculated based on a minimum soil unit weight of 100 pounds per cubic foot. The soils within the passive zone at the foot of retaining walls (one footing width in front of the wall to a depth equal to the footing depth) should be tested to verify that the soils have the minimum unit weight of 100 pounds per cubic foot (with moisture). If the soils have a unit weight of less than 100 pounds per cubic foot, the soils within this zone should be over-excavated and replaced as engineered fill. These soils should be tested prior to backfilling behind the wall.

8.6.4 A minimum factor of safety of 1.5 should be used when combining the frictional and passive resistance of the soil to determine the total lateral resistance. The upper 12 inches of subgrade should be neglected in determining the total passive resistance.

8.6.5 The active and at-rest pressures of the native soils and engineered fill may be assumed to be equal to the pressures developed by a fluid with a density of 50 and 75 pounds per cubic foot, respectively. These pressures assume level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

8.6.6 The active and at-rest pressures were calculated based on a maximum soil unit weight of 135 pounds per cubic foot. The compacted soils behind the retaining walls should not have a compacted unit weight above 135 pounds per cubic foot (with moisture). If the soils have a unit weight of greater than 135 pounds per cubic foot, the soils should be over-excavated and replaced at a lower degree of compaction. If the backfill soils must be placed at a unit weight of over 135 pounds per cubic foot to achieve minimum compaction requirements the material should not be used as backfill behind retaining walls.

8.6.7 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

8.6.8 The wall designer should determine if seismic increments are required. If seismic increments are required, contact Twining for recommendations for seismic geotechnical design considerations for the retaining structures.

8.6.9 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a drain, either a filter fabric encased gravel section or a geo-composite drain, to prevent hydrostatic pressures from acting on the walls. Drainage should be collected in a perforated pipe which can carry drainage from behind the walls.

8.6.10 It is recommended that lighter hand operated or walk behind compaction equipment be used to compact soils within 5 feet of retaining walls to reduce the potential for wall damage during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure.

8.7 <u>Retaining Walls</u>

8.7.1 Retaining wall plans, if used and when available, should be reviewed by Twining to evaluate the actual backfill materials, proposed construction, drainage conditions, and other design geotechnical parameters.

8.7.2 Retaining walls should be supported on spread or continuous footings placed entirely on at least 24 inches of engineered fill or engineered fill which extends to a minimum depth of 36 inches below preconstruction site grades, whichever is deeper. Footings should have a minimum width of 15 inches, regardless of load.

8.7.3 Spread and continuous footings may be designed for a maximum allowable soil bearing pressure of 2,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.7.4 Shallow spread footings should have a minimum depth of 24 inches below rough pad grades or adjacent exterior grades, whichever is lower.

8.7.5 Retaining walls should be constructed with non-expansive granular free-draining backfill placed within the zone extending from a distance of 1 foot laterally from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. This requirement should be detailed on the construction drawings. Granular backfill will reduce the effects of shrink and swell on the wall.

8.7.6 Segmented wall design (mechanically stabilized walls) should be conducted by a California licensed geotechnical engineer familiar with segmented wall design and having successfully designed at least three walls at sites with similar soil conditions. None of the data included in this report should be used for wall design. A design level geotechnical report should be conducted to provide wall design parameters. If the designer uses the data in this report for wall design, the designer assumes the sole risk for this data.

8.7.7 Retaining walls may be subject to lateral loading from pressures exerted from the soils, groundwater, slabs-on-grade, and pavement traffic loads, adjacent to the walls. In addition to earth pressures, lateral loads due to slabs-on-grade, footings, or traffic above the base of the walls should be included in design of the walls. The designer should take into consideration the allowable settlements for the improvements to be supported by the retaining wall.

8.7.8 Retaining walls should be constructed with a drain system including at least drain pipes surrounded by at least 5 cubic feet of crushed ¾ inch or ½ inch rock backfill fully encapsulated in Mirafi 140 N, or equivalent. Drain pipes near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary)

8.8.2 Interior floor slabs should be supported on 6 inches of Class 2 aggregate base (AB) over 12 inches of non-expansive fill over engineered fill extending to a depth of at least 24 inches below the deepest foundation bearing grade, or at least 36 inches below preconstruction site grades, or at least 12 inches below improvements to be removed, whichever provides the deeper fill. The AB should comprise the top of the non-expansive section. The Class 2 AB material is recommended for structural purposes, to provide a capillary break, and to provide a working surface during construction. In addition, the engineered fill should be tested and should be within optimum moisture content and three (3) percent above the optimum moisture content prior to placement of the base course. If the moisture content does not meet these requirements, the engineered fill should be moisture conditioned and re-compacted immediately prior to placement of the AB section.

8.8.3 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

8.8.4 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarding membrane when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. We recommend that Stegowrap 15 or equivalent should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The layer of Stegowrap 15 should overlay a minimum of 4 inches of compacted Class 2 AB. It should be noted that placing the PCC slab directly on the vapor barrier will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor barrier directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Mr. Eric Gerst with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the Class 2 AB and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. We recommend the membrane be selected in accordance with ASTM C 755-02, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to ASTM E 154-99 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier selection and installation conform to the ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R-96), Addendum, Vapor Retarder Location and ASTM E 1643-98, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under to preclude hydrostatic surcharge to the wall in the event the storm drain system did not function properly. Clean out and inspection points should be incorporated into the drain system. Drainage should be directed to the site storm drain system.

8.7.9 The fill side of retaining walls should be fully covered with a Miradrain material, or as shown on the project plans, whichever is more stringent (highest water carrying capacity).

8.7.10 If open graded materials such as crushed rock are used as drain material, these materials should be fully encased in filter fabric and compacted to a non-yielding condition under the observation of the geotechnical engineer. A Caltrans Class 2 permeable material, installed without the use of filter fabric, is preferable to open graded material as it presents a lower potential for clogging than the filter fabric. Class 2 permeable material should be compacted to 95 percent relative compaction in accordance with ASTM D1557.

8.7.11 The contractor should use light hand operated or walk behind compaction equipment in the zone equal to one wall height behind the wall to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure. The contractor is responsible for damage to the wall caused by improper compaction methods behind the wall.

8.7.12 If retaining walls are to be finished with dry wall, plaster, decorative stone, etc., waterproofing measures such as manufactured drainage boards (i.e., Miradrain 6000 or 6200 or approved alternative) should be applied to moisture proof the exterior of the walls. Waterproofing should also be used if effervescence (discoloration of wall face) is not acceptable. The waterproofing system should be designed by a qualified professional.

8.8 Interior Slabs-on-Grade

The recommendations provided herein are intended only for the design of interior concrete slabs-on-grade and their proposed uses, which do not include construction traffic (i.e., cranes, concrete trucks, and rock trucks, etc.). The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

8.8.1 The floor slabs should be reinforced for the anticipated temperature and shrinkage stresses. A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slabs-on-grade for a differential vertical movement (total and differential) of the floor slabs of ½-inch in 40 feet horizontal distance.

Concrete Slabs. In addition, it is recommended that the manufacturer of the floor covering and floor covering adhesive be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements.

8.8.5 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc.) should be caulked per manufacturer's recommendations.

8.8.6 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations. Once repaired, the membrane should be inspected by the contractor and the owner to verify adequate compliance with manufacturer's recommendations.

8.8.7 The manufacturer's requirements vary regarding the surface and cover material around the placed membrane. Vapor retarding membranes should be installed in accordance with the manufacturers' specifications.

8.8.8 The vapor retarding membrane is not required beneath exposed concrete floors, such as warehouses and garages, provided that moisture intrusion into the structure are permissible for the design life of the structure.

8.8.9 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings (such as wood or vinyl). These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.45 lb./lb. or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or flatwork adjacent to the structure, 4) providing adequate drainage away from the structure, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structure.

8.8.10 The moisture vapor transmission through the slab should be tested at a frequency and method as specified by the flooring manufacturer. Vapor transmission results should be within floor manufacturer's specifications prior to placing flooring.

8.8.11 To avoid damaging slabs during construction, the following recommendations are presented: 1) design for the anticipated settlements presented in this report; 2) provide at least 6 inches of aggregate base below the slabs; and 3) design for anticipated construction equipment loading such as cranes. If cranes are to be used, the contractor should provide slab loading information to the slab design engineer to determine if the slab is adequate.

8.8.12 If tilt wall construction is used, a perimeter pour strip between the wall footing and the adjacent interior slab should be incorporated into the project design. After the walls are erected and a majority of the differential movement has occurred, the pour strip should be placed.

8.8.13 Backfill the zone above the top of footings at interior column locations, building perimeters, and below the bottom of slabs with an approved backfill and/or an aggregate base section as recommended herein for the area below interior slabs-on-grade. This procedure should provide more uniform support for the slabs which may reduce the potential for cracking.

8.8.14 If near surface soils are subjected to precipitation or moisture conditioned to above about 5 to 6 percent above optimum moisture content, these soils may be unstable during grading; and therefore, could require stabilization. Stabilization could include a geotextile fabric and aggregate base backfill, and/or chemically treating the unstable soils. The contractor is responsible for stabilization of unstable soils due to moisture.

8.8.15 To provide a design modulus of subgrade reaction of 150 psi/in, the slabs should be supported on a minimum of 6 inches of Class 2 aggregate base material (R-value of 78). In addition, if concrete trucks will be traveling over the aggregate base material or the aggregate base will be used as a working surface, the contractor should determine an adequate aggregate base section thickness for the type and methods of construction proposed for the project. The aggregate base section may be included in the non-expansive engineered fill recommended below the floor slabs. The proposed compacted subgrade can experience instability under high frequency concrete truck loads during slab construction resulting in heaving and depressions in the subgrade during critical pours. This condition becomes more critical during wet winter and spring months. A layer of AB can reduce the potential for instability under the high frequency loading of concrete trucks. The improved support characteristics of the AB can be used in the design of the slab sections. Therefore, it is recommended to utilize a slab design with at least 6 inches of AB for constructability purposes and structural purposes.

8.9 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic. These recommendations are intended for rather lightly loaded sidewalks, curbs, and planters, etc. Recommendations for concrete slabs subjected to vehicular traffic are included in a later section of this report.

8.9.1 Exterior concrete slabs-on-grade should be underlain by four(4) inches of aggregate base over eight (8) inches of non-expansive engineered fill over engineered fill to a depth of 12 inches below the slab or 18 inches below the preconstruction site grade, whichever is greater. In areas where exterior concrete slabs-on-grade are anticipated, the exposed ground surface to receive the slabs should be over-excavated to a depth of at least 12 inches below the slab level, at least 18 inches below preconstruction site grades, or at least 12 inches below improvements to be removed, whichever provides the deeper fill. The exposed subgrade soils should then be scarified to a depth of 8 inches, moisture conditioned, and compacted. The over-excavation should then be backfilled with engineered fill soils which have been moisture conditioned and compacted to a minimum of 92 percent of the maximum dry density as determined by ASTM Test MethodD1557. Exterior concrete slabs-on-grade should be include deepened edges to act as cutoff for lateral migration of moisture. The deepened edges should extend to a depth of 4 inches below the non-expansive section, or to a minimum of 12 inches below the bottom of the slab, whichever is greater.

8.9.2 If the subgrade is prepared, and then disturbed by equipment workers, weather or another source, we recommend that the exposed subgrade to receive slabs be tested to verify adequate compaction. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted to a minimum of 92 percent of the maximum dry density as determined by ASTM Test Method D1557. This condition should be verified prior to installation of plumbing, footing excavation, and construction of the slabs-on-grade.

8.10 Asphaltic Concrete (AC) Pavements

8.10.1 It is recommended that pavement sections be underlain by at least 12 inches of engineered fill, engineered fill extending to at least 18 inches below preconstruction site grades, or to at least 12 inches below improvements to be removed, whichever provides the deeper fill. The exposed subgrade soils should then be scarified to a depth of 8 inches, moisture conditioned and compacted to a minimum of 92% relative compaction prior to placing engineered fill to grade. Any soft or unstable areas identified during compaction of the bottom of the over-excavation should be removed and compacted as engineered fill. Prior to placement of fill, proof rolling, under the observation of Twining, should be performed. The zone of over-excavation should include all the pavement areas and extend laterally a minimum of three (3) feet beyond the edge of pavements or curbs.

8.10.2 The 12 inches of subgrade soils below the pavement section (asphaltic concrete and aggregate based) should be compacted to at least 95 percent relative compaction (ASTM D-1557).

8.10.3 The following pavement sections are based on an R-value of 17 and traffic index values ranging from 5.0 to 8.0. Traffic indices for the project should be selected by the project Civil Engineer. A design professional should select the appropriate pavement section based on the anticipated truck traffic expected for the site. It should be noted that if pavements are constructed prior to the building construction, the selected traffic index values may be too low and need to be increased. If the pavements are placed prior to construction, or if more frequent truck traffic is anticipated the design professional who selected the pavement sections should be contacted to re-evaluate the traffic index values.

Traffic Index	AC thickness (inches)	AB thickness, (inches)	Compacted Subgrade (inches)
5	3	8	12
5.5	3	9.5	12
6	3	11	12
6.5	3.5	12	12
7	3.5	13.5	12
7.5	4	14.5	12
8	4.5	15	12

AC - Asphaltic Concrete
AB - Aggregate Base compacted to at least 95 percent relative compaction (CAL test 216)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)

8.10.4 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended at least four (4) inches below the aggregate base section into native subgrade soils. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

8.10.5 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement section should be re-evaluated for the changed subgrade conditions.

8.10.6 If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic.

8.10.7 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

8.10.8 Pavement materials and construction method should conform to Sections 25, 26, and 39 of the State of California Standard Specification Requirements.

8.10.9 The asphaltic-concrete should be compacted to an average relative compaction of 97 percent, with no single test value being below a relative compaction of 95 percent based on a 50-blow Marshall maximum density.

8.10.10The asphalt concrete should comply with Type "B" asphalt concrete as described in Section 39 of the State of California Standard Specification Requirements. We recommend that an asphalt concrete mix design be prepared and approved prior to construction.

8.10.11It is recommended that the base course of asphaltic concrete consist of a 3/4" maximum, medium gradation. The top course, or wear course should consist of a ½" maximum, medium gradation. Mix designs should be provided to the owner and Twining for review and approval prior to placement of asphaltic concrete. All asphaltic concrete should be compacted as noted above.

8.11 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland cement concrete pavement structural sections are presented in the following subsections. The PCC pavement design assumes a minimum modulus of rupture of 550 psi. The design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic (both trucks and forklifts).

8.11.1 In new PCC pavement areas, the exposed subgrade should be over-excavated to at least 18 inches below the bottom of the proposed aggregate base layer. The exposed soils should be scarified, moisture conditioned, and compacted as engineered fill, and the pavement areas filled to pavement subgrade elevations as engineered fill. The upper 12 inches of subgrade soils should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.11.2 The "light duty" pavement section was designed based on an Average Daily Truck Traffic (ADTT) of three 5-axle trucks and forklifts per day (equivalent axial loads of 15 per day). A design k-value of 150 psi/in was used assuming 6-inches of Class 2 aggregate base material (minimum R-value of 78) is placed over the compacted native soils (the estimated k-value of the native soils equals 70 psi/in).

Pavement Component	Thickness, Inches
Portland Cement Concrete	6.5
Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Compacted Subgrade (95% Minimum Relative Compaction)	12.0 minimum

8.11.3 The "heavy duty" pavement section was designed based on an ADTT of 30 trucks and forklifts, and a k-value of 150 psi/in was used assuming 6-inches of Class 2 aggregate base material (minimum R-value of 78) is placed over the compacted native soils (the k-value of the native soils equals 70 psi/in).

Pavement Component	Thickness, Inches
Portland Cement Concrete	7.0
Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Compacted Subgrade (95% Minimum Relative Compaction)	12.0 minimum

8.11.4 For interior or exterior slabs subject to non-pneumatic tire forklift traffic, a PCC section should be designed by others considering the loads, tire patterns, spacing etc. A design modulus of subgrade reaction of 70 psi/in should be used for slabs supported on native soils (R-value 17). A design k-value of 150 psi/in was used assuming 6-inches of Class 2 aggregate base material (minimum R-value of 78) is placed over the compacted native soils (the estimated k-value of the native soils equals 70 psi/in). This assumes that subgrade and base materials are compacted as recommended in the PCC section provided.

8.11.5 It is suggested to place a geotextile fabric of Tensar BX1200 or Mirafi BASX Grid 12, or equivalent, on a compacted subgrade below the AB section for PCC pavement sections. A geotextile fabric would help prolong the life of the pavements by preventing fine grained subgrade soils from migrating into the AB section.

8.11.6 Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.

8.11.7 Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet by 12 feet for a 6-inch slab thickness. Regardless of slab thickness, joint spacing should not exceed 15 feet.

8.11.8 Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.

8.11.9 Control joints should have a depth of at least one-fourth the slab thickness, e.g., 1.5-inch for a 6-inch slab.

8.11.10 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas. Construction joint location should be determined by the contractor's equipment and procedures.

8.11.11Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

8.11.12Pavement construction should conform to Sections 40 and 80 of the State of California Standard Specifications.

8.11.13Fine grained native and engineered fill soils may become unstable during grading; and therefore, could require stabilization. Stabilization may include placing a geotextile fabric and aggregate base materials, and/or chemical treatment (i.e., chemical treatment) or a combination of these to stabilize soils. For bidding purposes for chemical treatment, 5 percent by weight high calcium quick lime should be used. Laboratory testing is not required for lime treatment intended for subgrade stabilization purposes.

8.12 Temporary Excavations

8.12.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability.

8.12.2 Temporary excavations should be constructed in accordance with CAL OSHA requirements. Temporary cut slopes should not be steeper than 1½ to 1, horizontal to vertical, and flatter if possible. If excavations cannot meet these criteria, the temporary excavations should be shored.

8.12.3 Shoring systems, if used, should be designed by an engineer with experience in designing shoring systems and registered in the State of California.

8.13 Utility Trenches

8.13.1 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional compliance with the manufacturer's requirements, governing requirements and this report, whichever is more stringent. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent. The width of the trench should provide sufficient space between the sidewall of the trench and the pipe to allow testing with a nuclear density gage (minimum 12 inches). However, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) ASTM C-33 sand. The bottom of the trench should be compacted prior to placement of the pipe bedding. The haunches and initial backfill (12 inches above the top of pipe) should consist of ASTM C-33 sand that is placed in maximum 6-inch thick lifts compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be non-expansive material compacted to a minimum of 92 percent relative compaction. All materials should be placed at between one (1) percent and four (4) percent above optimum moisture content. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.13.2 If ribbed or corrugated pipes are used on the project, then the backfill should extend to at least 1 foot above the top of pipe or as required by the manufacturer, whichever is greater, to prevent damage to the pipe by the compaction operations above the pipe. Crushed gravel should be used below (bedding) and around the pipe and should be entirely encased in an approved geotextile fabric such as Mirafi 140 N or equivalent. However, a geotextile fabric would not be required if the granular materials consist of Caltrans Class 2 Permeable material. In either case, the sand, gravel, and/or Class 2 Permeable material should be densified using both vibratory and compaction equipment to achieve a non-yielding condition and a minimum relative compaction of 92 percent. The haunches should be hand tamped to achieve the required relative compaction. The maximum lift thickness shall be 6 inches unless approved in writing by Twining. The backfill within the pipe zone should be a crushed gravel material placed and compacted in a manner to fill the irregular exterior surface of the pipe. The gravel should be compacted to a non-yielding condition under the observation of Twining. As an alternative, the pipe zone can be backfilled with a sand-cement slurry.

8.13.3 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be moisture conditioned to between one (1) percent and four (4) percent above the optimum moisture content and compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

8.13.4 On-site soils and approved imported engineered fill may be used as final backfill in trenches.

8.13.5 Jetting of trench backfill will not be allowed to compact the backfill soils.

8.13.6 Where utility trenches extend from the exterior to the interior limits of a building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

8.13.7 Storm drains and/or utility lines should be designed to be "watertight." If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil heave causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. It is recommended that the pipelines be inspected and pressure tested prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are "watertight."

8.13.8 The plans should note that utility trench backfill for electrical lines, irrigation lines, etc. should be compacted to a minimum relative compaction of 95 percent per ASTM D1557.

8.13.9Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 2 horizontal to 1 vertical downward from the bottom of building foundations.

8.13.10The project Civil Engineer should include slurry type cutoff collars along utility trenches at critical locations to prevent the migration of surface water into the trench and along the trench backfill material.

8.13.11Granular soils and approved imported engineered fill may be used as final backfill in trenches provided they meet the approved project plans and specifications.

8.14 Corrosion Protection

8.14.1 Based on the ASTM Special Technical Publication 741 and the analytical results of one (1) soil sample analysis, the soil has a "mildly corrosive" corrosion potential to ferrous alloy pipes, as indicated by a resistivity value of 16,000 ohms/centimeter and a pH value of 5.8. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on the "mildly corrosive" corrosion potential of the soil. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated.

8.14.2 A low potential for corrosion of concrete due to sulfate attack is anticipated based on the 0.00068 concentrations (by dry weight) of sulfates determined for the near-surface soils. According to Table 19-A-3 of the CBC, this concentration of sulfates falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. The ACI Manual of Concrete Practice, Section 201.22-12, recommends using Type I and II cement for foundations placed in these soils.

8.14.3 These soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Twining does not practice corrosion engineering.

9.0 DESIGN CONSULTATION

9.1 Twining should be provided the opportunity to review those portions of the contract drawings and specifications that pertain to earthwork operations, slabs-on-grade, pavement areas, and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is part of this current contractual agreement.

9.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

9.3 If Twining is not afforded the opportunity for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Twining.